Design aspects for monopile foundations

Aspects du dimensionnement pour les fondations monopieux

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ABSTRACT: This paper describes the outcome of a recently completed research project – known as PISA – on the development of a new process for the design of monopile foundations for offshore wind turbine support structures. The PISA research was concerned with the use of field testing and three-dimensional (3D) finite element analysis to develop and calibrate a new one-dimensional (1D) design model. The resulting 1D design model is based on the same basic assumptions and principles that underlie the current *p-y* method, but the method is extended to include additional components of soil reaction acting on the pile, and enhanced to provide an improved representation of the soil-pile interaction behaviour. Mathematical functions – termed 'soil reaction curves' – are employed to represent the individual soil reaction components in the 1D design model. Values of the parameters needed to specify the soil reaction curves for a particular design scenario are determined using a set of 3D finite element calibration analyses. The PISA research was focused on two particular soil types (overconsolidated clay till and dense sand) that commonly occur in north European coastal waters. The current paper provides an overview of the field testing and 3D modelling aspects of the project, and then focuses on the development, calibration and application of the PISA design approach for monopiles in dense sand.

RESUME : Ce papier décrit les résultats d'un projet de recherche terminé récemment – connu sous le nom de PISA – portant sur le développement d'une nouvelle procédure pour le dimensionnement des fondations monopieux pour les structures support des éoliennes offshore. La recherche PISA s'est intéressée à l'utilisation d'essais in-situ sur pieux et d'analyse (3D) par éléments finis afin de développer et de calibrer un nouveau modèle de dimensionnement unidimensionnel (1D). Le modèle de dimensionnement 1D obtenu est basé sur les mêmes hypothèses et principes fondamentaux à la base de la méthode p-y actuelle. Cependant, la méthode est étendue afin d'inclure des composants supplémentaires à la réaction du sol sur le pieu, et améliorée afin de fournir une meilleure représentation du comportement de l'interaction sol-pieu. Des fonctions mathématiques – appelées 'soil reaction curves' (courbes de réaction du sol) – sont utilisées afin de représenter les composants individuels de la réaction du sol dans le modèle de dimensionnement 1D. Les valeurs des paramètres requis pour spécifier les courbes de réaction du sol pour un scenario de dimensionnement donné, sont déterminés au moyen d'un ensemble d'analyses de calibration 3D par éléments finis. La recherche PISA s'est concentrée sur deux types de sols spécifiques (argile glaciaire surconsolidée et sable dense) communément présents dans les eaux côtières du nord de l'Europe. Le présent papier offre une vue d'ensemble des caractéristiques des essais in-situ sur pieux et des modélisations 3D du projet, et se concentre ensuite sur le développement, la calibration et l'application de l'approche de dimensionnement PISA pour les monopieux dans du sable dense.

KEYWORDS: monopile design, 1D model, soil reaction curves

1 INTRODUCTION

The monopile is the dominant foundation system for current and planned offshore wind farm developments in shallow coastal waters, particularly in Europe. Monopile foundations in this application are typically designed with the aid of simplified analysis approaches, such as the '*p-y*' method and its variants, in which the foundation is modelled as an embedded beam, with the lateral load-displacement interaction between the soil and pile represented by non-linear functions known as *p-y* curves. Simplified computational models of this sort facilitate the development of multiscale optimisation procedures in which models of the performance of individual wind turbine structures are employed within a computational framework to optimise an entire windfarm. For a wind turbine foundation model to be useful in this context, it must be fast to compute and sufficiently accurate and reliable for design purposes. Analyses based on the *p-y* method can be computed rapidly, but current forms of

the *p-y* method – which have their origins in the design of long, relatively flexible piles – are widely regarded as being unreliable for the design of monopiles with relatively small values of *L*/*D* (where *L* is embedded length and *D* is pile diameter), see e.g. Doherty and Gavin 2011.

A recent project – known as PISA (**PI**le **S**oil **A**nalysis) – employed field testing and computational modelling to develop a new design approach for monopile foundations for offshore wind turbine applications. In this new approach, the underlying simplicity of the *p-y* method – in which the pile is modelled as an embedded beam – is retained, but additional components of soil reaction are incorporated within the design model to improve its performance.

The PISA project was conducted, between 2013 and 2016, as a joint industry/university study. The scientific programme was developed by an Academic Working Group (AWG) with members drawn from Oxford University, Imperial College and University College Dublin. The research was supported by the project partners listed in the Acknowledgements section of this paper; Dong Energy acted as the lead partner and main contractor. PISA consisted of three related research strands: (i) reduced-scale field testing of monopile foundations (with associated site investigation and laboratory soil testing) (ii) three-dimensional (3D) finite element modelling and (iii) the development of a new one-dimensional (1D) modelling approach for design.

The PISA research was focused on monotonic lateral loading, as this was identified to be the area where substantial gains could be achieved. Although it is acknowledged that improved design methods for cyclic loading are needed, it is first necessary to have robust procedures for monotonic loading before addressing more complex cyclic loading issues. The methods developed in PISA are capable of future extensions to cyclic loading, and additional cyclic testing was conducted during the field testing phase to aid such extensions.

To limit the scope of the research the project was concerned specifically with two sets of soil conditions that are commonly encountered in north European coastal waters: (i) a stiff overconsolidated clay till and (ii) a dense to very dense marine sand. At an early stage in the project, potential sites for the field tests were sought, with approximately homogeneous profiles consisting of each of these two soil types. The outcome of this process was that a site at Cowden in the north east of England was selected as the clay site and a site at Dunkirk in northern France was selected as the sand site.

The current paper provides an overview of the design approach that was developed during the PISA project, together with a description of the process that was employed to calibrate the 1D design model for a representative offshore soil profile consisting of dense sand. The development of representative soil conditions and the model calibration process were based on the results of the Dunkirk field tests and associated 3D finite element modelling.

A similar process to calibrate the design model for a representative offshore clay till site (not discussed in the current paper) is described in Byrne et al. 2017.

2 THE PISA DESIGN APPROACH

2.1 *Formulation of the PISA design model*

The components of the PISA design model are illustrated in Figure 1. A monopile foundation is represented in the model as an embedded beam with moment M_G and horizontal force H_G applied to the pile at the ground surface. Four separate components of soil reaction are assumed to act on the embedded monopile. Consistent with the standard *p-y* method, a

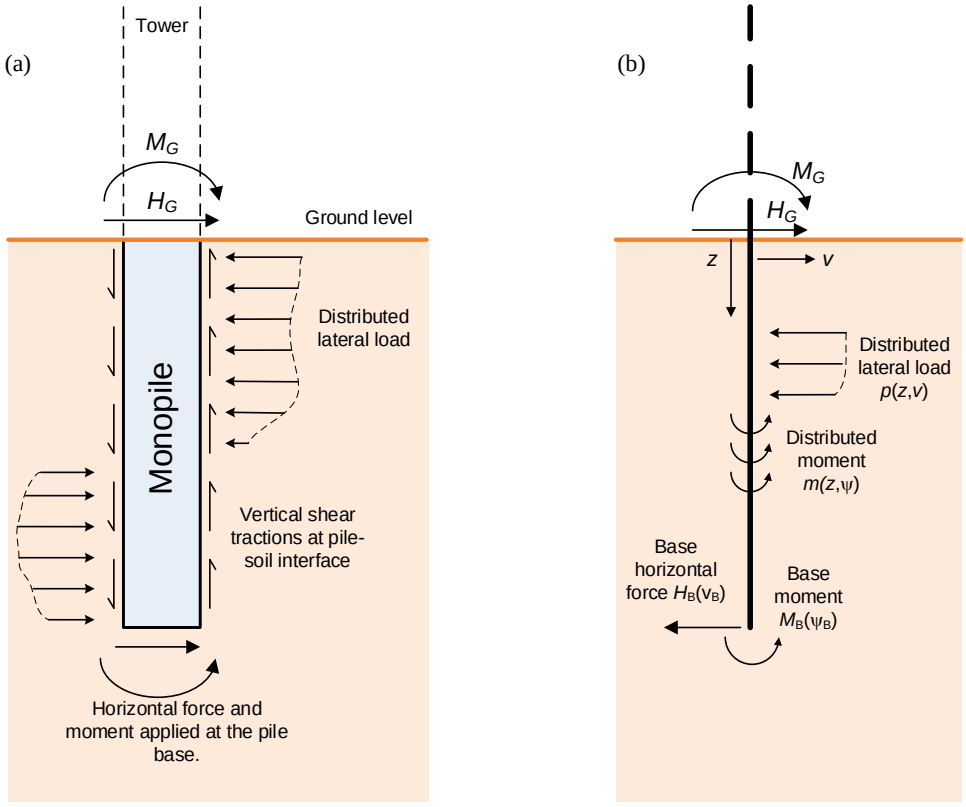


Figure 1. PISA 1D monopile model (a) assumed soil reactions acting on the monopile (b) 1D design model. In the left figure, the soil reactions are shown in the directions that they are likely to act, given the applied loads that are indicated. In the right figure, the indicated directions of the soil reactions are consistent with the cordinate directions shown.

distributed lateral load, p (units of force / length) acts on the pile. Additionally, a distributed moment, m (units of force×length / length) is applied; this distributed moment is caused by the vertical tractions that are induced at the soil-pile interface when local pile rotations occur, as indicated in Figure 1a and Figure 2.

A horizontal force H_B and a moment M_B acting on the base of the pile are also included in the design model. A four-component model of this sort has previously been employed for the design of drilled shafts for onshore applications (e.g. Lam 2013) and has previously been described in the context of the PISA research by Byrne et al. 2015a.

In the current implementation of the model, the monopile is represented by Timoshenko beam theory, which allows the shear strains in the pile to be incorporated in the analysis in an approximate way. Since the influence of the shear strains on the overall pile deformation is likely to increase as L/D is reduced, the use of Timoshenko theory provides a means of maintaining the robustness of the approach as the embedded length of the monopile reduces (or the diameter is increased).

Consistent with the conventional *p-y* method, the soil reactions are applied to the embedded beam on the basis of the Winkler assumption, i.e. in which the p and H_B components are specified to be functions only of the local pile displacement, v, and m and M_B are specified to be functions only of the local pile cross-section rotation, ψ. Functions relating the soil reactions and the local pile displacements (or rotations) are termed 'soil reaction curves'. Although the Winkler approach neglects the coupling that inevitably occurs between adjacent soil layers, it provides a convenient basis for design calculations, as demonstrated by the widespread adoption of the *p-y* method.

It should be noted, however, that soil reaction curves determined on the basis of the Winkler approach are unlikely to be unique. Appropriate soil reaction curves may depend, for example, on the relative magnitude of the translational and rotational movements of the pile. It is desirable, therefore, to calibrate the soil reaction curves using pile deformation modes that are representative of those that are expected to be experienced by full-scale wind turbine monopile foundations.

The PISA design model reduces to the standard *p-y* approach when m, H_B and M_B are set to zero (and appropriate choices are made on the relationship between p and the local lateral pile displacement, v). Experience has shown, however, that m, H_B and M_B become increasingly important as L/D is reduced (Byrne et al. 2015a). The distributed moment component, for example, depends on the pile diameter; it increases in magnitude as the pile diameter is increased. Similarly the force and moment reactions H_B and M_B at the base of the pile become increasingly significant as L/D is reduced. The four-component model in Figure 1 therefore provides a rational way of addressing a feature of the *p-y* method, that has come to be known as the 'diameter effect', in which the standard *p-y* curves (e.g. API 2010, DNV 2016) are typically found to become increasingly unreliable as the pile diameter is increased, or the pile length is reduced (e.g. Alderlieste et al. 2011, Doherty and Gavin 2011, Lam 2013).

The design model in Figure 1 has been implemented as a 1D finite element model adopting a Galerkin formulation. The implementation is based on standard procedures for non-linear finite element analysis. The functions representing the soil reaction curves are embedded in the 1D model (in much the same was as a constitutive model is embedded in a standard 3D finite element program) with parameters specified by the user.

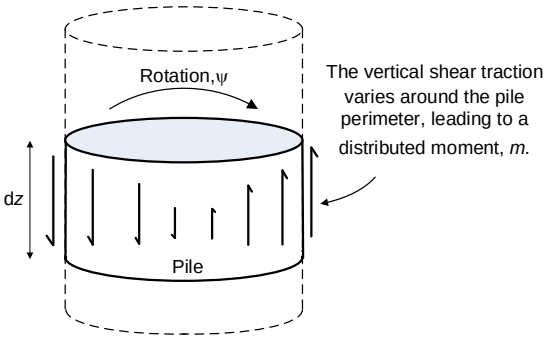


Figure 2. Diagrammatic view of the vertical tractions acting at the soil-pile interface for an elemental length of pile, dz, These tractions are assumed equivalent to a distributed moment reaction, *m*.

2.2 *Selection and calibration of the soil reaction curves*

The functions selected to represent the soil reaction curves are, to an extent, arbitrary. They should, however, be capable of representing the soil reactions for behaviour ranging from small displacements (needed, for example, to predict the dynamic response and natural frequencies of a wind turbine structure) to the large displacement response (required for the calculation of the Ultimate Limit State, ULS). The conventional cube root function for *p-y* curves in clay (DNV 2016) is unsuitable (unless modifications are introduced) since it implies an infinite initial stiffness. The two-parameter hyperbolic tangent function typically adopted for *p-y* curves in sand (DNV 2016) employs parameters to control both the initial stiffness and the ultimate value of the distributed lateral load. However, it is not possible to tune this function to match the shape of the response between these two limits, or to specify the magnitude of displacement needed to mobilise the ultimate value of distributed load.

In the current work, a four-parameter conic function – described in further detail later – is employed for each of the soil reaction curves in the design model. This four-parameter function appears to provide a reasonable compromise between ability to represent the soil reactions at an appropriate level of detail and the desirability (from a practical perspective) of minimizing the total number of parameters in the model.

Ideally, the soil reaction curves employed in the PISA design model would be calibrated directly, using the results of field testing on full-sized monopiles. However, conducting experiments on full-sized structures would be prohibitively expensive and it would be impractical to devise a test programme that encompasses all of the relevant soil, pile and loading parameters. Also, considerable technical difficulties exist in devising instrumentation systems to determine the various soil reaction components from the measured pile performance.

In PISA, an alternative approach was adopted, in which the calibration of the soil reaction curves in the 1D design model was related, indirectly, to the results of the field testing campaigns conducted during the project, as illustrated in Figure 3. This model calibration process, which employed three separate activities (field testing, 3D finite element modelling

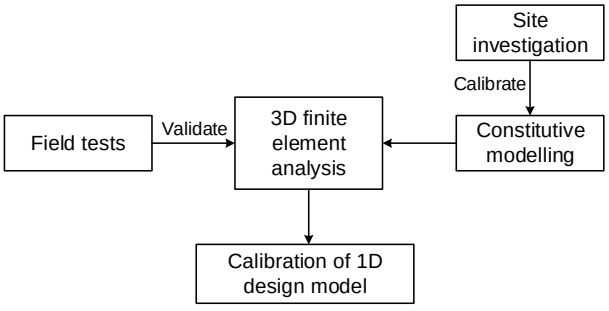


Figure 3. PISA design model development process.

and 1D model development) is summarized in general terms below.

Field tests involving lateral loading of monopiles were conducted, at a reduced scale, at the two selected test sites (Cowden and Dunkirk). Bespoke 3D finite element models were developed for several of the field test piles. The constitutive models that were selected for the analyses, to reproduce the behaviour of the two soils, were calibrated using the results of advanced site investigations employing pre-existing data combined with intensive new laboratory and field investigations conducted during the PISA project (Zdravković et al. 2015). Data obtained from the field tests were compared with the 3D finite element results to confirm the veracity of the 3D modelling procedures; this process is indicated as 'validate' in Figure 3. A key feature of this validate process is that artificial means of achieving a match between the numerical model and the field tests were rigorously rejected.

Once the 3D finite element simulations of the field tests had been completed, a separate 3D finite element parametric study (referred to below as the 'calibration analyses') was conducted. These calibration analyses adopted homogeneous clay and sand profiles that were based on the soil conditions at each of the two test sites, but adjusted to obtain soil profiles that are representative of offshore conditions. The soil at the onshore Dunkirk test site, for example, was interpreted to have a (small) suction near the surface. Since this is unrepresentative of submerged offshore conditions, a surface layer with negative pore pressures was not included in the 3D finite element calibration analyses employed to develop the PISA sand model. The calibration analyses adopted a range of pile geometries and loading conditions that were judged to span the likely design space for full-sized monopiles.

Numerical data from the 3D finite element calibration analyses were used, directly, to calibrate the soil reaction curves employed in the PISA 1D model. This approach provides an indirect link between the field test data and the 1D model, via the use of 3D finite element modelling procedures. Data processing activities that would be infeasible for the field data (such as extracting the individual soil reaction curves from the soil-pile interface tractions) can be conducted straightforwardly when applied to the 3D finite element results.

The current paper provides a description of the application of this process to calibrate the PISA 1D model for a homogenous sand profile with relative density D_R = 75% based on the Dunkirk field tests, employing the procedures indicated in Figure 3. The paper also demonstrates the application of the 1D model to two design cases with pile parameters that differ from those employed in the 3D finite element calibration process.

3 DUNKIRK FIELD TESTING CAMPAIGN

3.1 *Site details*

The Dunkirk test site is located in a coastal area in northern France, near to the town's Port Ouest. Earlier laboratory and field research described by Chow 1997, Kuwano 1999, Jardine et al. 2006, Ahgakouchak 2015 and Ahgakouchak et al. 2015 showed that the site consists principally of a dense Flandrian sand with a surface layer (about 3m thick) of dense hydraulically-placed sand that has the same geological origin as the deeper Flandrian deposit. New CPTu and seismic cone soundings conducted for PISA and new advanced laboratory tests described by Liu et al. 2017 show that the hydraulic fill and Flandrian sand layers of the soil present relative densities of 100% and 75% respectively, with a critical state friction angle of $\phi'_{cs} = 32^o$. The laboratory tests provide extensive information on the sands stress-dilatancy behaviour and highly non-linear stiffness characteristics.

3.2 *Testing details*

Monopiles with diameter D = 0.273 m, 0.762 m and 2.0 m, and values of L/D between 2 and 10 were installed at the test site. The piles were instrumented with a range of above and below ground instrumentation, including inclinometers, optical fibre Bragg grating strain gauges and extensometer strain gauges. Testing was conducted using the arrangement shown diagrammatically in Figure 4. The test piles were loaded at a height h (termed 'load eccentricity') above ground via a hydraulic ram reacting against a reaction pile. Most of the field tests employed a protocol in which the applied horizontal load, H, was controlled to apply a constant ground-level velocity of D/300 per minute to the pile. At various stages during these constant velocity tests, the applied load was held constant, to allow observations to be made of time-dependent behavior (e.g. due to creep and/or consolidation). A test was considered complete when (i) the ground-level pile displacement exceeded $v_G = D/10$ and (ii) the ground-level pile rotation exceeded 2°. A few tests were conducted at elevated displacement rates and a limited amount of cyclic testing was also conducted. The 3D finite element models of the field tests, however, were validated only with respect to the constant velocity tests. Further details of the field testing campaign are given in Byrne et al. 2015b.

4 3D FINITE ELEMENT MODELLING

4.1 *3D finite element analysis of the Dunkirk tests*

3D finite element models were developed for several of the test piles at the Dunkirk site. Analyses were conducted using the finite element program ICFEP (Potts and Zdravković 1999), adopting the modelling procedures outlined in Zdravković et al. 2015.

The chosen constitutive model was a bounding surface plasticity model (Taborda et al. 2014). This is a critical state model within the state parameter framework, capable of reproducing the stress level- and void ratio-dependent behaviour of sands. The model was calibrated using the earlier research by Imperial College, combined with recent triaxial test data on Dunkirk sand reported in Aghakouchak 2015 and Aghakouchak et al. 2015 as well as the work conducted for PISA described by Liu et al. 2017. The interface between the pile and the soil was represented by an elasto-plastic Mohr-

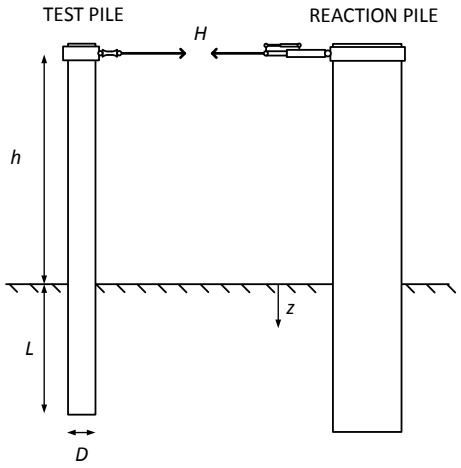


Figure 4. Illustration of the pile testing system employed at the Cowden and Dunkirk sites.

Coulomb model with zero cohesion and a friction angle of 32°.

Detailed consideration of the near-surface ground conditions indicated that the 3 m thick hydraulic fill layer developed higher CPT q_c resistances than had been found in the mid-to-late 1990s investigations described by Chow 1997 and Jardine et al. 2006. It was postulated that the additional shallow resistance was a result of either ageing in the hydraulic fill, local spatial variations, light cementation between the sand grains, or of suctions due to partial saturation. Detailed investigation to fully explain these variations was not feasible. However, piezocone tests did show clear signs of suctions developing at shallow depths and the analyses therefore employed a realistic limited suction (i.e. pore pressures less than hydrostatic) in the soil above the level of the water table (estimated to be at a depth of 5.4 m below ground level). It is noted that the additional complexities involved in modelling unsaturated surface layers are absent from offshore sites, where soils are typically assumed to be fully saturated.

Example field data for a D = 0.762 m, L = 4 m pile (identified as pile DM4) tested at the Dunkirk site, are shown in Figure 5. Figure 5a shows the relationship between the applied lateral load H and the lateral displacement of the pile at ground level, v_G. The constant load steps in the field data indicate the hold periods that were prescribed during the testing process. Also shown in Figure 5a are an unload-reload loop conducted soon after the start of the test, and the unloading response of the pile at the end of the test. The response computed using the 3D finite element analysis is also shown in Figure 5a; this shows good agreement with the field data. Comparisons of a similar quality were obtained for the field test data and the corresponding finite element models for the two large diameter (D = 2 m) piles tested at the site.

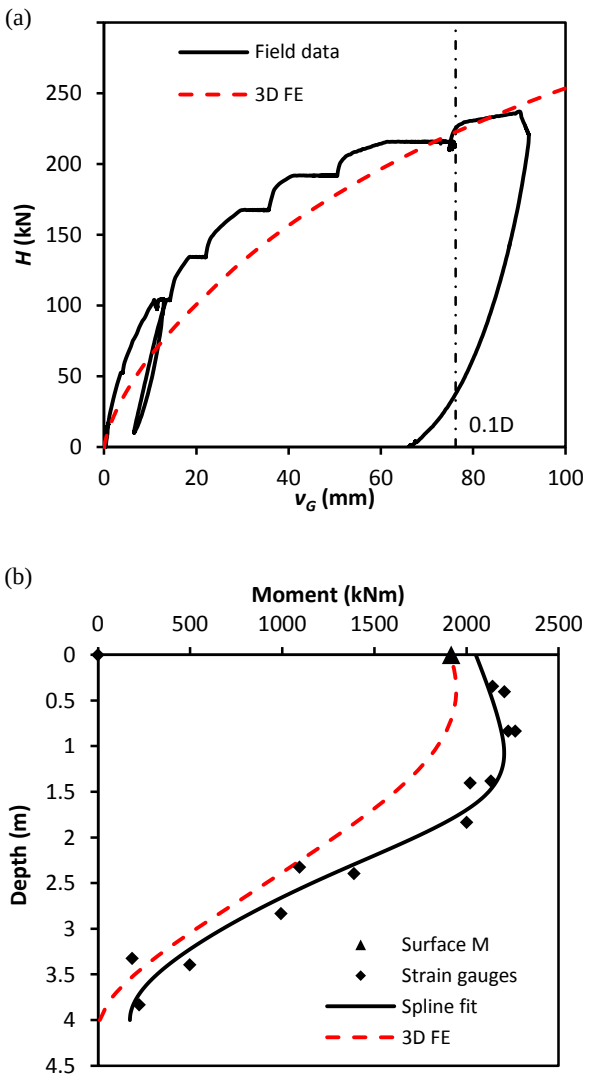


Figure 5. Comparisons between the field measurements and 3D finite element results for pile DM4 (D = 0.762 m, L = 4 m) tested at Dunkirk; (a) horizontal load, H, vs. ground displacement, v_G, response (b) below-ground bending moments at $H = 192$ kN, $v_G = 41$ mm. In (b) 'Surface M' indicates the bending moment determined from the value of the horizontal load, 'Strain gauges' refers to bending moments inferred from the fibre optic Bragg grating strain gauges, 'Spline fit' indicates a spline that has been fitted to the bending moment data.

Figure 5b shows the measured bending moments induced in pile DM4, deduced from the strain gauge instrumentation, at the start of the hold period for $H = 192$ kN. The distribution of bending moments determined from the 3D model at this same value of applied horizontal load is also shown in the figure. The agreement between the two sets of data appears reasonable.

The finite element results were found to be less consistent with the field data for the shortest D = 0.762 m pile (with L = 2.3 m) that was tested at this site. For this relatively short pile, the finite element results are highly dependent on the initial conditions that are assigned to the unsaturated surface layer. Since some uncertainty existed on the appropriate conditions to apply to this surface layer (as a consequence of limitations in the available site investigation data) the finite element results are regarded as being less robust for this particular pile.

The broadly satisfactory comparison between the field test data and the numerical analysis supports the use of the 3D finite element model to calibrate the soil reaction curves in the 1D design model, as described below.

4.2 *3D finite element calibration study*

The calibration analyses were based on the soil conditions at the Dunkirk test site, together with the constitutive model and associated constitutive parameters that were employed in the finite element analyses of the field tests. Certain adjustments to the Dunkirk soil profile were required, however, to ensure that the profile employed in the calibration analyses was representative of an offshore homogeneous sand site. In particular, in the calibration study, hydrostatic pore pressures were assigned to the entire soil profile (noting that a region with a small suction above the water table was adopted to model the Dunkirk field tests). Although the Dunkirk site profile shows variations in sand state with depth, uniform relative density of

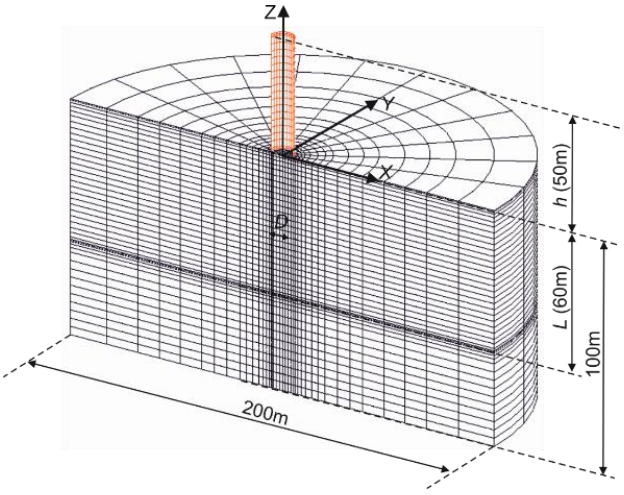


Figure 6. 3D finite element mesh for pile C4 (D = 10 m, L = 60 m, h = 50 m) in the calibration analysis set.

75% was assigned to the soil for these calibration analyses.

Eleven calibration analyses were conducted for monopile dimensions and load eccentricities in the range $5\text{m} < D < 10\text{m}$, $2 < L/D < 6$, $5 < h/D < 15$. A typical mesh employed for the calibration analyses is shown in Figure 6.

5 1D MODEL DEVELOPMENT FOR SAND

5.1 *Soil reaction curves*

Numerical representations of the soil reaction curves (referred to below as 'numerical soil reaction curves') were determined from the 3D finite element calibration analyses using a process in which nodal forces acting at the soil-pile interface, and stresses in the interface elements between the pile and the soil, were extracted. Numerical data on the distributed lateral load, *p*, were determined by integrating the *x*-components (using the coordinate system in Figure 6) of the horizontal tractions acting on the pile at discrete values of depth along the pile. Data on the distributed moment were obtained by integrating the vertical soil-pile tractions, accounting for the distance between the pile neutral axis and the point on the pile perimeter where the traction is applied. The force and moment reactions at the pile base were obtained by integration of the stresses in the layer of soil elements immediately below the pile base.

To proceed, it is necessary to use appropriate non-dimensional parameters to process the numerical results and also to formulate the soil reaction curves. The non-dimensional forms employed for the PISA sand model are listed in Table 1, where σ'_{vi} is the local value of initial vertical effective stress, *G* is the local value of small-strain shear modulus, v and ψ are the local pile lateral displacement and cross-section rotation respectively. The normalization process for the distributed moment follows a pattern that differs from the other three soil reaction components. It was identified, when reviewing the numerical soil reaction curves, that the distributed moments appeared to scale conveniently with the current value of the local distributed lateral load, *p*. Since the vertical tractions induced on the pile arise as a consequence of friction at the soil-pile interface, it seems plausible that the magnitude of *m* is correlated with the normal tractions applied to the pile. The normal tractions are, themselves, closely associated with the distributed lateral load, *p*; it therefore seemed appropriate in the current modelling to normalize the distributed moment, *m*, with the local value of the distributed load, p. The use of the non-dimensional form $\bar{m}$ in Table 1 implies that the distributed moment *m* is a product of the current value of *p* and a separate function of the local rotation, ψ. Although this adds to the complexity of the 1D model, this form of soil reaction curve is incorporated straightforwardly within the 1D finite element formulation used in the model.

Table 1. Parameter normalization.

Normalized variable	Non-dimensional form
Distributed lateral load, $\bar{p}$	$\dfrac{p}{\sigma'_{vi}D}$
Lateral displacement, $\bar{v}$	$\dfrac{vG}{D\sigma'_{vi}}$
Distributed moment, $\bar{m}$	$\dfrac{m}{pD}$
Pile rotation, $\bar{\psi}$	$\dfrac{\psi G}{\sigma'_{vi}}$
Base shear load, $\bar{H}_B$	$\dfrac{H_B}{\sigma'_{vi}D^2}$
Base moment, $\bar{M}_B$	$\dfrac{M_B}{\sigma'_{vi}D^3}$

The soil reaction curves employed in the 1D model, referred to below as the 'parametric soil reaction curves' are formulated in terms of the normalized variables listed in Table 1. The four-parameter conic function used to represent the soil reaction curves is illustrated in Figure 7, where $\bar{x}$ signifies a normalized displacement or rotation variable and $\bar{y}$ signifies the corresponding normalized soil reaction component. The conic function is calibrated by the specification of four parameters $(k, n, \bar{x}_u, \bar{y}_u)$, each of which has a straightforward interpretation. The parameter k specifies the initial slope; $\bar{y}_u$ is the ultimate value of the normalized soil reaction and $\bar{x}_u$ is the normalized displacement (or rotation) at which this ultimate value of soil reaction is reached. The parameter n $(0 < n < 1)$ determines the shape of the curve.

Values of the parameters defining the soil reaction curves for each of the soil reaction components were determined via an automatic optimization process conducted over the complete set of eleven 3D finite element calibration analyses. In conducting this optimization, the soil reaction curve parameters for $\bar{p}$ and $\bar{m}$ were assumed to vary linearly with depth along the pile.

Initial values for the model parameters were determined by least-squares fitting of the numerical soil reaction curves, moderated by eye. The calibration was further improved by allowing adjustments to these parameters to optimize the fit between the *H* vs v_G responses computed using the 3D finite element model and the 1D model for $0 < v_G < 0.1D$.

5.2 *Example soil reaction curves*

Two contrasting examples of the process of fitting the four-parameter conic function to the data extracted from the 3D finite element calibration analyses are shown in Figure 8 and Figure 9.

Figure 8 shows example data on normalized distributed lateral load, $\bar{p}$, at various pile depths, *z*, for a calibration calculation (pile C4) with *D* = 10 m, *L* = 60 m. At shallow depths, where soil displacements are relatively large, a peak is typically found to occur in the response, followed by post-peak softening. An example of this type of response, for $z/D = 0.23$ is shown in Figure 8a. This behaviour is closely linked to the dilational characteristics of the soil. Since softening behavior cannot be represented with the four-parameter conic function employed in the 1D model, it was necessary to make an arbitrary choice on the ultimate value $\bar{p}_u$ of the normalized distributed load for incorporation in the model. In the current work, when post-peak softening was observed, $\bar{p}_u$ was taken as an intermediate value between the peak and final values determined from the 3D finite element calibration data. The resulting parametric soil reaction curves (determined by optimizing over the complete set of eleven calibration analyses) are plotted for a range of depths, *z*, for the full range of soil displacements developed in the calibration analysis in Figure

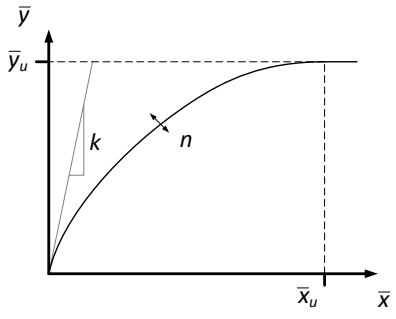


Figure 7. Four-parameter conic function employed to represent the soil reaction curves.

8a, and for small displacements in Figure 8b. Differences are shown to exist between the data from the 3D calibration analysis and the resulting calibrated soil reaction curves. As the parametric curves are tailored to provide a representation of the 3D finite element data across the complete set of calibration analyses, they therefore can sometimes exhibit the tendency, apparent in Figure 8, to depart significantly from the 3D calibration data for individual piles at a local level.

In contrast, Figure 9 indicates the match between the parametric soil reaction curve and the numerical data for the base moment for pile C4; in case of this component of soil reaction, the two data sets are seen to agree well.

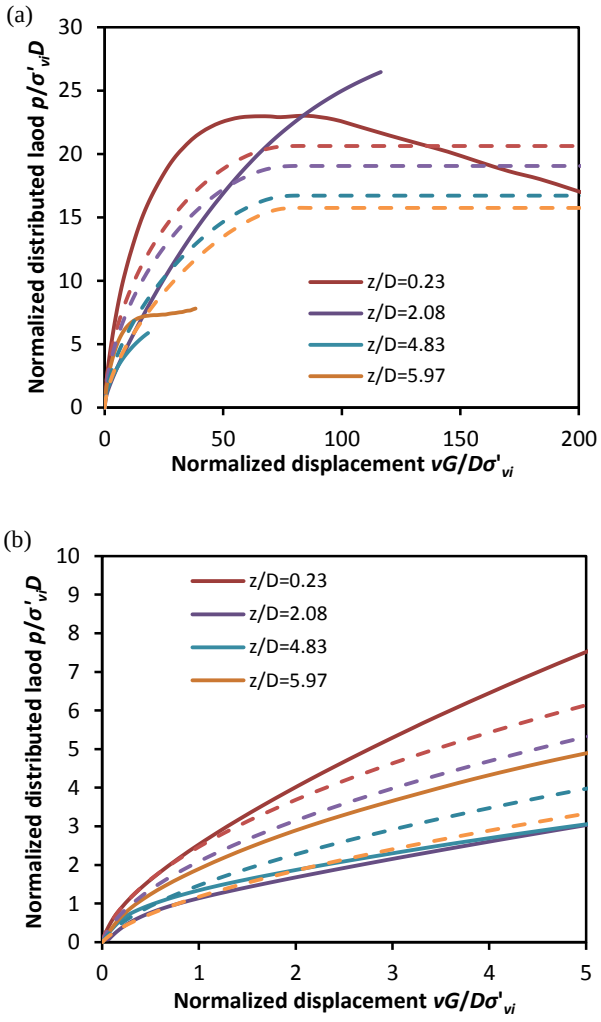


Figure 8. Example soil reaction curves for normalized distributed lateral load for calibration pile C4 (D = 10 m, L = 60 m). Solid line are data from the 3D calibration analysis of the pile, dashed lines are the calibrated soil reaction curves; (a) large displacements, (b) small displacements.

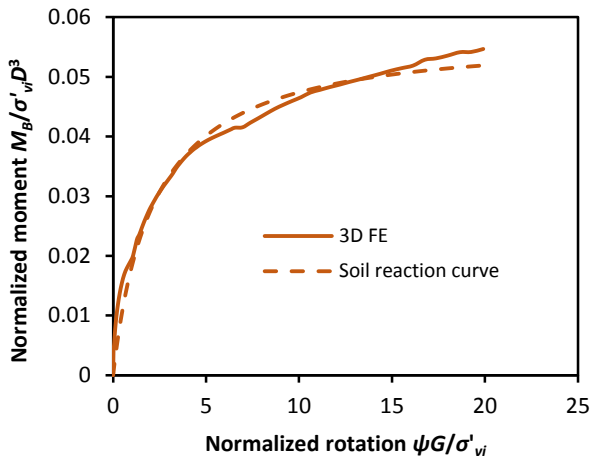


Figure 9. Parametric soil reaction curve (shown as a dashed line) and 3D finite element calibration data (shown as a solid line) for the base moment M_B, for pile C4 (D =10 m, L = 60 m).

5.3 *1D model analysis of the calibration cases*

Example comparisons of the computed H vs v_G response for two of the D = 10 m diameter calibration piles are shown in Figure 10; pile C1 has length 20 m and pile C4 has length 60 m. In spite of the apparently poor performance of the four-parameter function in representing aspects of the computed lateral soil reaction curves for pile C4, as shown in Figure 8, the overall performance of the 1D model – in terms of the extent to which 1D predictions of the H vs v_G response agree with the 3D calibration data for piles C1 and C4 – is seen to be excellent. Similarly close comparisons between the 1D and 3D results were obtained for all of the other calibration analyses.

This exercise indicates that the 1D model is able to reproduce the overall behavior of the calibration piles, although at a local level, significant differences can exist between the

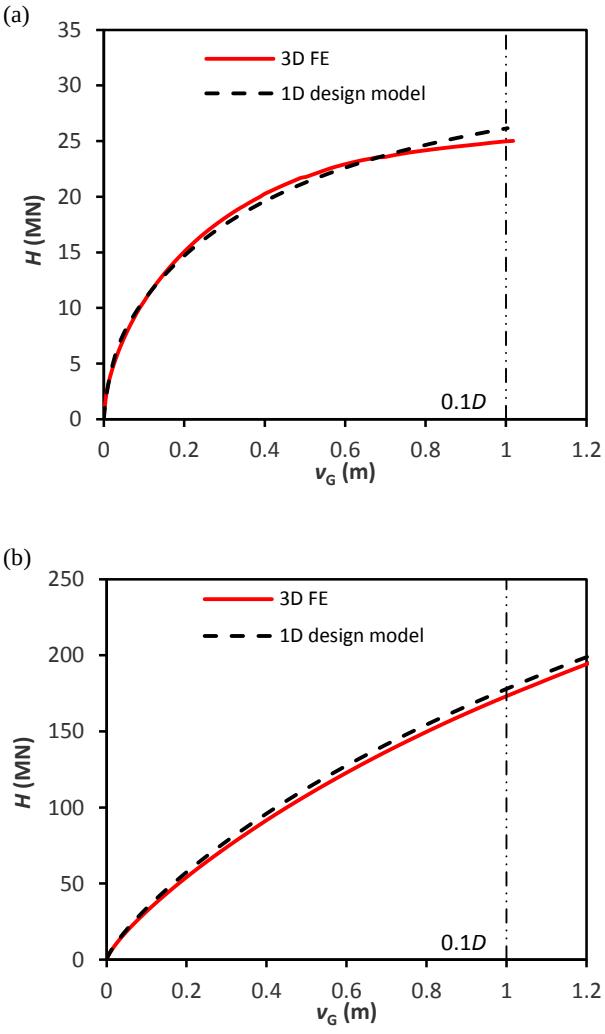


Figure 10. Comparisons between the 3D finite element calibration analyses and the 1D model computed response (a) for pile C1 (D = 10 m, L = 20 m) (b) for pile C4 (D = 10 m, L = 60 m).

calibration data and the parametric soil reaction curves. This well-conditioned aspect of the 1D model is considered to be due to the overall performance being obtained by integrating the soil reaction curves along the entire length of the foundation. Provided that significant systematic errors are absent from the 1D model, the averaging process employed in the modelling procedure appears to have the consequence that the model is remarkably tolerant of imperfect fitting of the data at a local level.

6 DESIGN EXAMPLE

Once the 1D model has been calibrated – or 'trained' – it can be used to determine the performance of a monopile foundation for arbitrary values of geometry and loading parameters that lie within the calibration space. To demonstrate the predictive capability of the model, two separate example design analyses have been considered. The geometric configurations adopted for these test cases, specified in Table 2 (where t is pile wall thickness), were selected to fall within the parameter space adopted for the calibration analyses, as indicated in Figure 11.

Table 2. Pile parameters selected for the two design examples.

Reference	D (m)	H (m)	L (m)	t (mm)
D1	7.5	37.5	22.5	68
D2	8.75	8.75	35	91

The load-displacement responses computed using the 1D model and subsequently the 3D finite element model for both of the design examples, D1 and D2, are shown in Figure 12. The results indicate a close match between the two sets of data over the full range of applied loading (up to a ground level pile displacement of $v_G = 0.1D$). Figure 13 shows the lateral displacements induced in the embedded portions of the piles for an applied horizontal load of $H = 0.75\,H_{max}$, where H_{max} is the value of horizontal load at $v_G = 0.1D$ determined from the 3D finite element analysis. The two sets of data are seen to agree well. The close agreement between the computed responses obtained using the 3D and 1D models for these two design cases is consistent with the assumption implicit in the PISA methodology, that the 1D model provides an efficient means of interpolating the overall pile response computed using the 3D calibration calculations to other pile geometries within the calibration space.

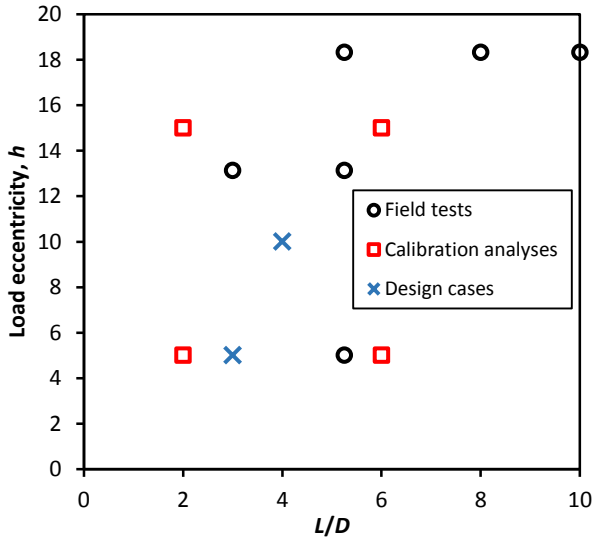


Figure 11. Parametric geometry space for the field tests, the calibration analyses and the design examples.

7 DISCUSSION

A method has been presented to calibrate a 1D model of monopile behaviour using a suite of 3D calibration analyses. The calibration process has been demonstrated for an example offshore site where the soil is a uniform, dense sand. In this example, the 1D model is shown to provide a close representation of the overall pile behaviour for each of the calibration analyses. Comparing the 1D model with the data that were used to calibrate it does not, in itself, provide any evidence of its predictive capability. This comparison exercise does, however, indicate that the various approximations and assumptions inherent in the 1D model do not detract significantly from its reliability.

The predictive capability of the 1D model has been demonstrated by means of two independent design examples with pile parameters that differ from those employed in the calibration set (although within the bounds of the calibration space).

It is suggested that the PISA modelling approach could be

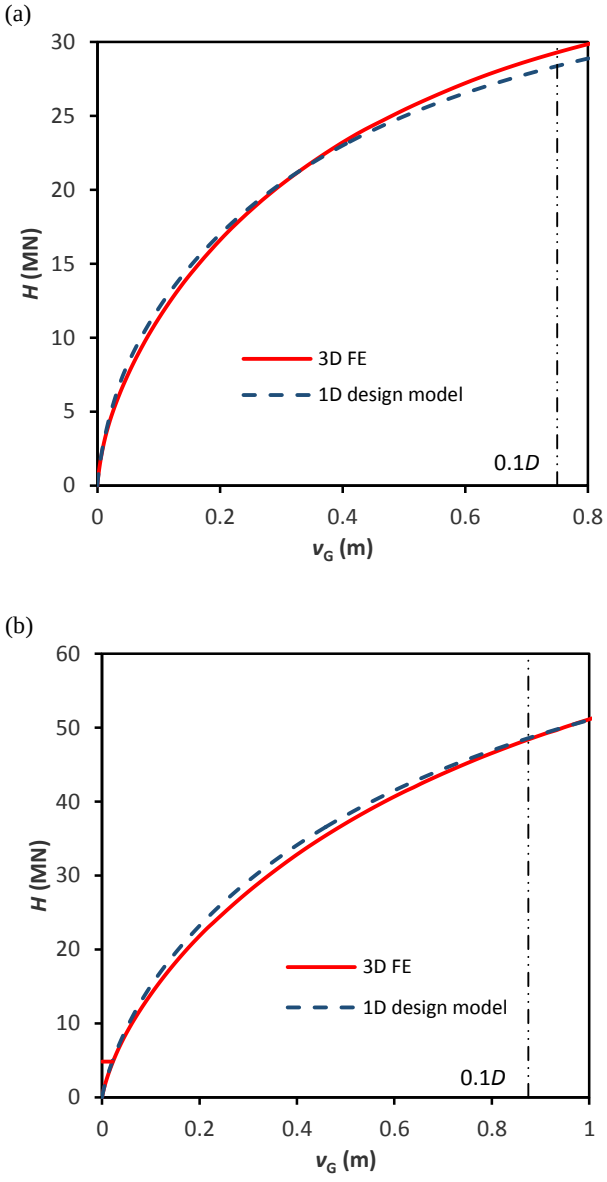


Figure 12. Comparisons between load-displacement responses computed using the 3D finite element model and the 1D model for the two design examples (a) D1 (b) D2.

employed for monopile design in one of two ways. For initial design calculations, it may be appropriate to employ pre-defined functions and parameters to represent the soil reaction curves based on the soil profiles established for any given site as characterized by relatively simple index or other testing. (This process is broadly similar to current forms of the *p-y* method). In the PISA terminology, this approach is referred to as the 'rule-based' method.

For more detailed and robust design calculations, an alternative route is proposed in which the site conditions are investigated more intensively with advanced sampling, laboratory and field techniques being applied to establish the detailed behavior of the principal soil types present at any given wind farm location. Bespoke 3D finite element calculations may then be performed that span the likely ranges of soil profiles and the parameter space that will control the final design. Soil reaction curves may then be extracted from the finite element analyses predictions for the soil-pile interface tractions. Procedures for this latter approach – which is termed the 'numerical-based' method – are demonstrated in the current paper for a homogeneous dense sand site. The numerical-based approach is essentially a procedure to train a relatively simple calculation (the PISA 1D model) using data from more detailed 3D finite element analyses. The 1D calculation is rapid to compute, with accuracy that is linked to the fidelity of the 3D finite element models that are employed in the calibration process. This approach means that full use can be made of any site investigation data (via the constitutive model employed in the 3D calibration analyses) in the formulation of the soil reaction curves. Moreover, the method can evolve with future developments in site investigation, constitutive modelling and finite element analysis.

The PISA modelling process is suitable for assessments of the ULS performance as well as for predicting the small-displacement dynamic performance of an offshore wind turbine structure. The current paper is limited to the modelling of monopile behaviour in a dense sand (although a similar calibration study, not reported here, has been completed for an overconsolidated clay till). Further development work is needed to extend the method to other soil types, included layered soils, and to include the effects of cyclic loading.

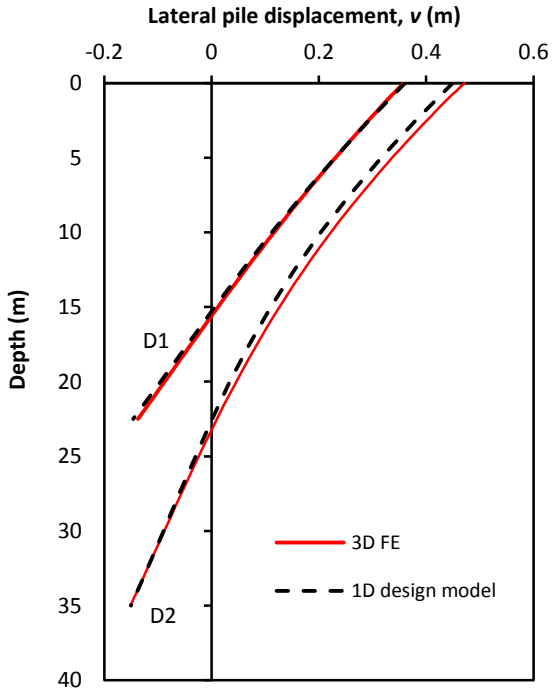


Figure 13. Below-ground lateral displacements for design examples D1 and D2 computed using the 3D finite element model and the calibrated 1D design model. The D1 and D2 results correspond to H = 22.1 MN and 36.15 MN respectively.

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